GOLDEN GATE BRIDGE and HIGHWAY DISTRICT

SAN FRANCISCO, CALIFORNIA

A REPORT ON A STUDY OF:

THE AMMANN & WHITNEY REPORT, AUGUST 1964; THE TUDOR ENGINEERING COMPANY REPORT, MARCH 1966: RELATING TO THE INSTALLATION OF A LOWER DECK FOR VEHICULAR TRAFFIC ON THE GOLDEN GATE BRIDGE

By
CLIFFORD E. PAINE
Consulting Engineer
Fennville, Michigan

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TABLE OF CONTENTS

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May 26, 1966

To the Honorable Board of Directors
Golden Gate Bridge & Highway District
Box 9000, Presidio Station
San Francisco, California

Gentlemen:

I am pleased to submit herewith my report on a study of the Ammann & Whitney Report dated August 1964, and the Tudor Engineering Company Report dated March 1966, outlining plans for the addition of a lower deck for vehicular traffic on the Golden Gate Bridge.

The scope of my study included only the suspension spans and their major carrying parts. The objective of the study was to determine which of the two plans would best serve the interests of the District.

The Ammann & Whitney plan with seven traffic lanes in the peak direction offers the most efficient use of the roadways without detracting from the appearance of the bridge or encroaching on its safety. This is the plan recommended for adoption.

In view of the certainty that a new crossing will not be built for many years the writer feels that the urgency of the traffic situation justifies you in expending from the reserve strength of the bridge the amount recommended. There will still remain a reserve capacity to take care of future additions up to five hundred pounds per lineal foot of bridge.

The writer appreciates the opportunity of again serving you and also appreciates the assistance received from General Manager, James Adam and his staff.

Respectfully submitted

Clifford E. Paine

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SUMMARY OF CONCLUSIONS

A study of the two plans proposed for the addition of a lower deck on the Golden Gate Bridge leads to the conclusion that the Ammann & Whitney Plan will best serve the interests of the Bridge District. The adoption and execution of this plan will not necessitate any objectionable modification of the bridge.

The appearance of the bridge will not be affected except for some flattening of the arched profile of the roadway between towers.

The reserve for future additional dead load amounting to 500 lbs. per ft. of bridge is sufficient for future essentials.

Unit stresses in the cables, towers, and anchorage chains are safe and the useful life of the bridge will not be shortened.

Under this plan seven lanes of traffic may be provided in the peak direction whereas provision is made for only five in the Tudor Engineering Plan.

The Tudor Engineering Plan requires abandonment of the existing bottom lateral system which has proven its effectiveness in stabilizing the bridge against torsional vibrations of high amplitude in extremely high winds. The lateral truss is needed.

Both plans involve much reconstruction of the approach structures. The Tudor Engineering Plan requires more extensive modification of the towers than does the Ammann & Whitney Plan.

The Ammann & Whitney Plan will be the less costly of the two by far since it provides all the required traffic relief without building a heavy lower deck roadway and without replacing the existing upper deck roadway slabs.

THE AMMANN & WHITNEY PLAN

GENERAL DESCRIPTION

The Ammann & Whitney Plan proposes the installation of a lower deck for automobile traffic. This deck would be placed just above the existing bottom lateral system. It would comprise an orthotropic steel fioor carried by a steel frame suspended from the existing floorbeams. The deck as proposed is of welded construction to be prefabricated and shipped to the site in sections convenient for shipment and erection.

This lower deck provides a roadway 44 ft. wide designed to carry four 11 ft. traffic lanes. An eight inch high steel curb, a service walk 2′6″ wide and a strong barricade are on each side of the roadway. The paving is a one and one-half inch thick special asphaltic compound.

The existing sidewalks remain as they are except for modification to reduce weight as hereinafter described.

As proposed by Ammann & Whitney the existing 60 ft. roadway would carry five lanes of mixed traffic operated with three lanes in the peak direction and two in the opposite direction. The four lane lower roadway would be operated either with two lanes in each direction or with four lanes in the direction of peak traffic.

STRUCTURAL CHANGES

Under the Amman & Whitney Plan no structural changes of any consequence are contemplated in the principal carrying members of the suspension spans. Some modifications must be made in the pylons, Marin anchorage housing and the approaches in order to provide clearance for the added

roadway. Such changes are intended to be made without interference with traffic on the existing roadway.

On the suspension spans the Interior Traveling Scaffolds will be removed as will also their track rails and rail chairs. Existing hangers supporting the transverse struts of the Bottom Lateral System will be replaced with new ones located closer to the center line of the bridge. No change is to be made in the Bottom Lateral System other than a shifting of the connections for these hangers.

Floor trusses and floorbeams adjacent to the pylons and towers will have to be modified to provide clearance for vehicles traveling on the lower deck.

At the towers the plan contemplates replacement of the upper halves of the diagonals of the tower bracing just below the upper deck. The new members will be placed at a slightly flatter angle with the horizontal.

The concrete slabs of the existing sidewalks will be replaced by metal plates with asphaltic wearing surfaces.

LOADS AND STRESSES

Design Dead Load

Ammann & Whitney arrive at a design dead load of about 25,100 lbs. per lin. ft. of bridge. This is made up as follows: —

Original Dead Load	21,300 lbs. per ft.
Added to date	1,600 lbs. per ft.
Lower Deck to be added	2,600 lbs. per ft.

A reduction in weight of sidewalks, removal of rails, etc., totaling about 1,150 lbs. per ft. combined with the figures tabulated above totals 24,350

lbs. per ft. To this they add 800 lbs. as a reserve for the future. The total is rounded out to 25,100 lbs. per lin. ft. of bridge.

The foregoing figures include a weight-saving of 130 lbs. per ft. of bridge effected by replacing the existing steel railings with ones of smaller sections made of aluminum. For reasons of appearance as well as economy it is believed that this change should not be made. The more rugged appearance of the existing railings seems appropriate. If it ever becomes necessary to do so the replacement of railings can be accomplished in the future just as easily. Without this change the weight-saving in sidewalks and from removal of track rails, etc., will total 1,100 lbs. per ft. Using this figure and reducing the reserve for future additions from 800 lbs. to 500 lbs. per ft. of bridge the design dead load would be 24,900 lbs. per ft. of bridge.

Design Live Load

For cables, anchorages and towers the design live load used is 2,900 lbs. per lin. ft. of bridge. This figure is broken down as follows:

400 lbs. per lane of upper deck	2,000 lbs.
175 lbs. per lane of lower deck	700 lbs.
100 lbs. on each sidewalk	200 lbs.
Total uniform live load per lin. ft.	2,900 lbs.

For the stiffening trusses the design live load is 3,250 lbs. per lin. ft. of bridge applied over restricted lengths.

The writer has no comment to make with respect to the above live loads other than to say that they are adequate. The figures are included here for convenient reference as too is the following statement of design live load for the deck structure and suspenders.

Design Load for Deck Structure and Suspenders

For the review of stresses in the existing members of the deck structure and the design of the lower deck Ammann & Whitney followed the Standard Specifications for Highway Bridges of the American Association of State Highway Officials (1961 Edition). They assumed five lanes of H20-S16 loading on the upper deck and four lanes of H10 loading on the lower deck.

Stresses in Cables

Ammann & Whitney report that for the loads assumed including five lanes of mixed traffic on the upper deck and four lanes of automobiles and other light vehicles only on the lower deck, the maximum axial stress in the cables will be about 91,500 lbs. per sq. in. In justifying this unit stress they point out that 90,000 lbs. per sq. in. was considered conservative on the Verrazano Narrows Bridge in New York having 12 lanes of unrestricted mixed vehicular traffic whereas in the case of the Golden Gate Bridge with the lower deck restricted to light vehicles the proportion of heavy trucks will probably be considerably less than on the Verrazano Narrows Bridge and hence a higher unit stress may be considered reasonably conservative.

The writer believes than an axial cable unit stress of 91,500 lbs. per sq. in. is too high. The fact that trucks are prohibited from use of the lower deck only means that they will utilize the upper deck and hence the overall effect will be unchanged. Using Ammann & Whitney's design dead load of 25,100 lbs. per ft. of bridge the writer finds for the fully loaded double deck bridge a maximum axial cable stress of about 90,150 lbs. per sq. in. Subsequent to the issuance of their report Ammann &Whitney have restated this unit stress to be 90,250 lbs. per sq. in. instead of 91,500 lbs. as originally reported.

With the design dead load reduced, as hereinbefore suggested, to 24,900

lbs. per ft. of bridge the maximum axial stress in the cables will be 89,600 lbs. per sq. in. which is acceptable.

Stresses in Anchorage Chains

Ammann & Whitney compute the maximum unit stress in the eye-bars of the anchorage chains to be 28,700 lbs. per sq. in. including temperature effects. They consider this to be well within safe limits in view of the fact that tests on full size eye-bars showed minimum yield point to be 50,000 lbs. per sq. in. and minimum ultimate strength to be 79,700 lbs. per sq. in.

The writer finds that under full live load at minimum temperature and with design dead load at 25,100 lbs. per ft. of bridge, the cable pull on the anchorage is 68,000,000 lbs. This produces a maximum axial stress of 28,400 lbs. per sq. in. in the eye-bars of the anchorage chains. The corresponding figures using the recommended design dead load of 24,900 lbs. per ft. of bridge, is a cable pull of 67,500,000 lbs. and maximum eye-bar axial stress of 28,200 lbs. per sq. in. This is considered acceptable.

The axial cable unit stress at the anchorage is 81,500 lbs. per sq. in. The anchorages are stable for this load.

Stresses in Stiffening Truss Chords

The critical stresses in the chords of the stiffening trusses caused by a combination of the dead load, a live load of restricted length, temperature and lateral wind force, are within the required limits. These maximum stresses are produced by combinations of temperature and dead load with either one-half live load and wind or with live load and one-half wind. For either of these combinations the allowable unit stress for silicon steel is

25,900 lbs. per sq. in. for compression on gross section and 32,000 for tension on net section.

The writer agrees with Ammann & Whitney that the stiffening trusses will not be over stressed by the addition of the lower deck as proposed by them.

Stresses in Tower Shafts

With a design dead load of 25,100 lbs. per ft. of bridge the fully loaded double deck structure at the specified minimum temperature will produce a cable reaction at the top of the tower of 67,670,000 lbs. At a silicon steel section eighty-three feet below the tower top the axial unit stress with the dead load of the tower included is 19,000 lbs. per sq. in., 5½% over that permitted by the original design specification.

Combinations of temperature and dead load with either one-half live load and transverse wind or with live load and one-half transverse wind should be evaluated. These forces produce a combination of axial and bending stresses for which a higher unit stress is allowed. The normal permissible unit stress allowed by the original design specifications was 24,000 lbs. per sq. in. for this stress condition on silicon steel.

Again at the section eighty-three feet below the top of the tower the combination of dead load plus one-half live load plus transverse wind at minimum temperature causes axial and bending stresses which total 24,900 lbs. per sq. in. The other combination which includes full live load with one-half transverse wind totals 24,100 lbs. per sq. in.

At a silicon steel section just below Strut No. 1 combined axial and bending stresses total about the same as those found at the section eighty-three feet below the top.

With the exception of two, all of the web plates of the tower shaft subject to the above stresses at the sections cited are built up to one and one-half inch thickness. The two exceptions are fifteen sixteenth inches thick.

The writer believes the towers are capable of sustaining the loads imposed on them by the double deck structure as proposed by Ammann & Whitney provided the future added dead load does not exceed 500 lbs. per ft. of bridge. This limits the design dead load to 24,900 lbs. per ft. of bridge and the live load of unrestricted length to 2,900 lbs. per ft. of bridge.

Stresses in Suspenders and Other Parts

The writer concurs with the Ammann & Whitney report that the suspenders and other parts of the suspension spans including piers, pylons, and anchorages are capable of sustaining the additional forces imposed on them by the addition of the lower deck.

SUPPLEMENTAL COMMENTS

After modification in accordance with the Ammann & Whitney Plan, the most efficient use of the roadways will be realized if three lanes of the upper deck and all four lanes of the lower deck are operated in the direction of peak traffic. The upper roadway could well be left six lanes as at present using three lanes in each direction and thus avoid the need for reversing traffic in the center lane of a five lane roadway.

It is suggested that the lower deck be limited to use by passenger automobiles and that "other light vehicles" be not included. This will result in better control.

THE TUDOR ENGINEERING PLAN

GENERAL DESCRIPTION

The Tudor Engineering Plan proposes the installation of a lower deck for mixed traffic. This deck would carry a roadway 60 ft. wide between curbs with a 10′ 6″ sidewalk and a suicide barrier on each side. The roadway is to be divided into five 12 ft. lanes for one way traffic.

The deck is carried on transverse floorbeams suspended below the bottom chords of the stiffening trusses and framed into the same at panel points. The roadway loads are distributed by steel grid type floor to longitudinal stringers and thence to the transverse floorbeams.

The plan contemplates performance of the work without at any time increasing the weight of the spans. This is to be done by discarding parts of the existing structure and also by substituting some parts of lighter weight. There is to be no interference with traffic.

STRUCTURAL CHANGES

Under this plan no changes are anticipated in the cables and anchorage blocks. Reinforcement of the tower shafts in the vicinity of the two roadways is contemplated in association with the removal of a portion of the upper cross of bracing and the placement of a structural substitute for the same. Also extensive changes are required at the ends of the center span adjacent to the towers.

Extensive structural changes are required at the pylons, Marin anchorage housing, and the approaches in order to provide clearance for the lower deck. Portions of both approaches must be reconstructed and some new structures are required for both.

The existing sidewalks with architectural handrails and electroliers are to be removed.

To make room for the framework supporting the roadway, the existing bottom lateral system is to be removed progressively.

The concrete slabs of the existing roadway are to be replaced by steel grid sections like the ones to be installed below. This will be done after the steel grid sections for the lower roadway are all installed but before they are paved.

LOADS AND STRESSES

Design Dead Load

The design dead load used is 21,680 lbs. per ft. of bridge. To this should be added 500 lbs. for filling up the slots in the pavement as hereinafter discussed, and another 500 lbs. for future additions. This brings the total design dead load to the rounded out figure of 22,700 lbs. per ft. of bridge.

Design Live Load

The design live load used consists of 4,200 lbs. per ft. of bridge for roadway loading and 580 lbs. per ft. of bridge for sidewalks. This makes a total of 4,780 lbs. per ft. of bridge for live load. It is believed that an overall loading of 4,500 per ft. would be conservative. This would result in a total of 2,950 lbs. per ft. per cable for the added dead load plus the live load.

Stresses in Cables

Using the live loads as reduced above the fully loaded bridge at the minimum specified temperature will produce cable stresses of about 87,800 lbs. per sq. in. This cable stress is acceptable.

Stresses in Anchorage Chains

Under full live load of 4,500 lbs. per ft., at minimum temperature and with design dead load of 22,700 lbs. per ft. of bridge the cable pull on the

anchorage is 66,200,000 lbs. and the maximum axial stress in the eye-bars of the anchorage chains is 27,700 lbs. per sq. in. This is acceptable.

Stresses in Stiffening Trusses

It is believed that the stiffening truss chords will not be overstressed. However, this should be investigated.

The stiffening truss verticals at alternate panel points where the suspenders are attached are not designed to carry floorbeam loads. No doubt these will need some modification in order to transfer loads from the lower floorbeams to the suspenders.

Stresses in Tower Shafts

The fully loaded double deck structure at the specified minimum temperature will produce a cable reaction at the top of the tower of 65,845,000 lbs. At a silicon steel section eighty three feet below the tower top the axial unit stress with the dead load of the tower included is 18,500 lbs. per sq. in. Combination of temperature and dead load with full live load and one-half transverse wind results in a total unit stress of 23,600 lbs. per sq. in. These stresses are acceptable.

Stresses in Suspenders and Other Parts

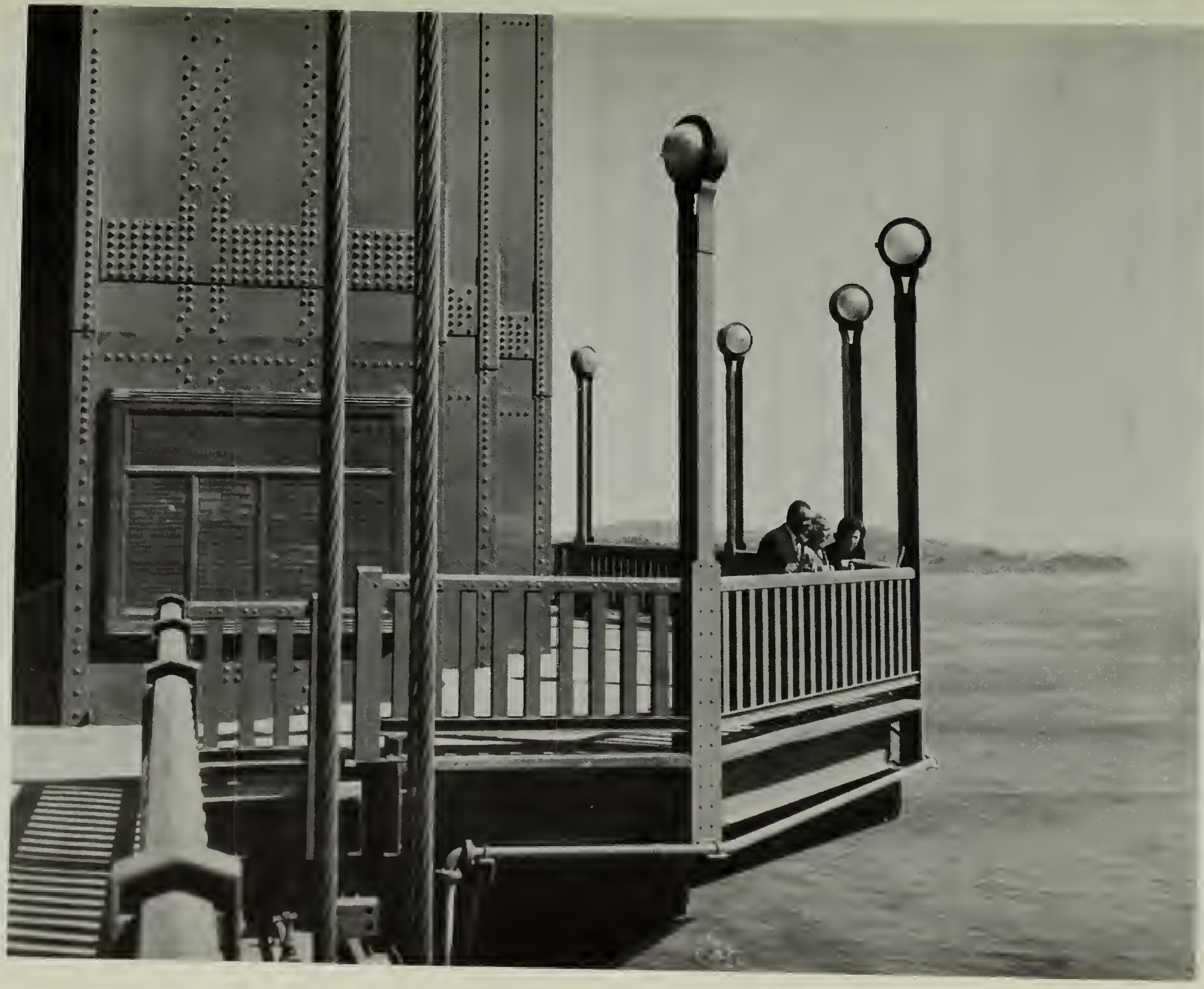
The writer believes that the suspenders and other parts of the suspension spans including piers, pylons, and anchorages are capable of sustaining the additional forces imposed on them by the addition of the lower deck.

DISCUSSION

Deck

No transverse expansion joints are provided in the lower deck. In a

Sidewalk around Tower



Viewing San Francisco from the South Tower

bridge of this type the writer believes that such expansion joints at intervals are necessary. The lengthening and shortening of bottom chords due to the flexure of the stiffening trusses is of such magnitude from one extreme to the other as to make this necessary.

Towers

The proposed removal of a part of the main material of the tower shafts and subsequent replacement of the same is extraordinary inasmuch as these shafts are under high axial stress from a load of over 60,000,000 lbs. at the top of each shaft. To remove and subsequently replace main material under a compressive unit stress approximating 13,000 lbs. per sq. in. would involve elaborate procedure and most careful planning.

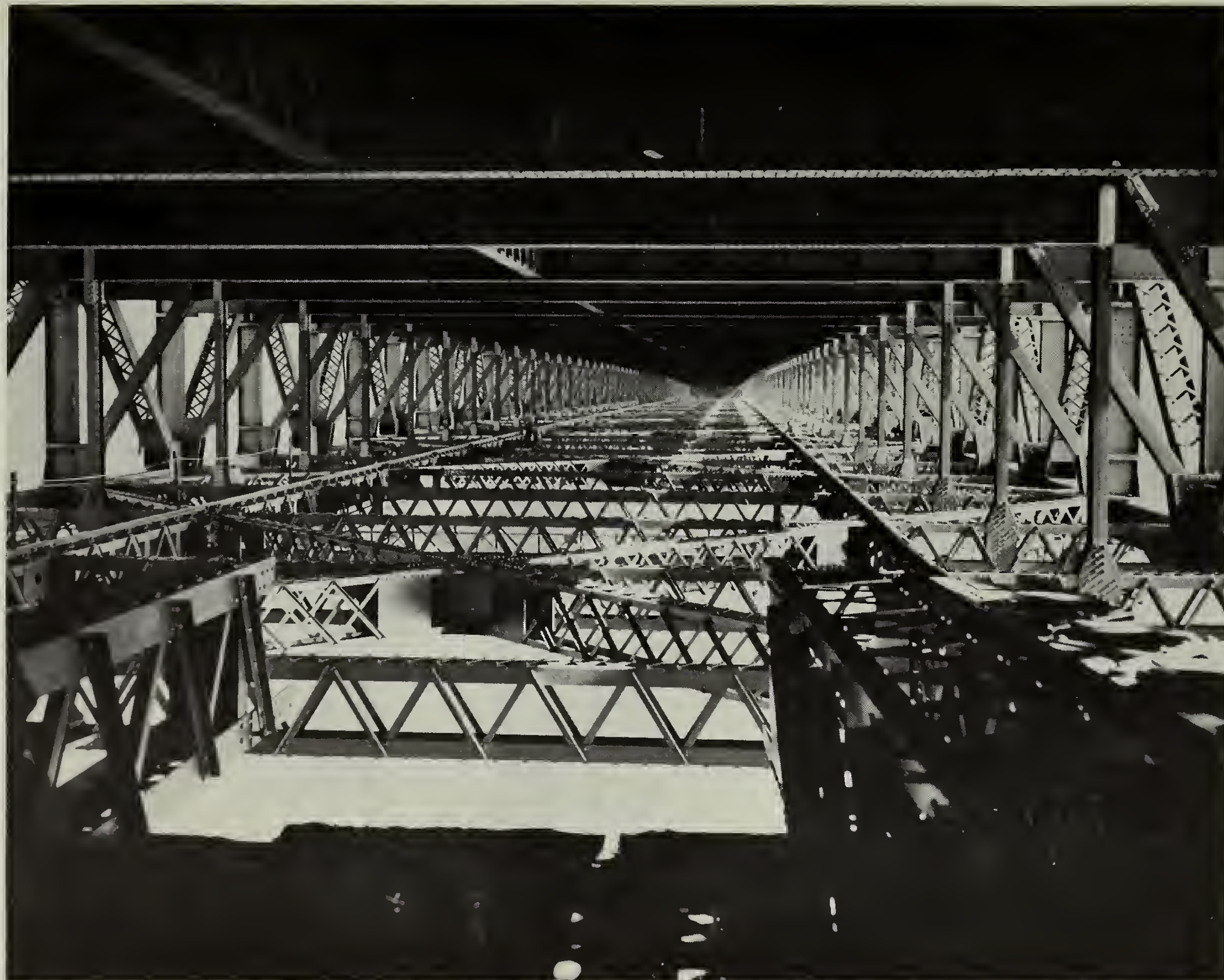
Sidewalks

The Golden Gate Bridge has been recognized all over the world for its architectural magnificence. It has become a symbol of San Francisco. It would be a sad mistake to downgrade the structure by removing the sidewalks, the architectural railing and electroliers.

Bottom Lateral System

The bottom lateral trusses were installed at a cost in excess of $3,000,000. Their sole purpose was to provide the suspended spans with great torsional resistance. They do not carry transverse wind loads to the towers. This is done by the top lateral system.

Before the bottom lateral trusses were installed a cross section through the suspended spans could be likened to an inverted U formed by the top laterals and the two stiffening trusses. By closing up the open side of the inverted U the bottom lateral truss changes the cross section from a three sided figure to a closed, four sided figure. In order to function properly it is essential that



The Bottom Lateral Truss

the bottom chords of the stiffening trusses form the chords of the bottom lateral trusses just as the top chords of the stiffening trusses form the chords of the top lateral truss. It is also essential that the cross sections at panel points be maintained rectangular. The kneebraces do this effectively. Since the system was installed over ten years ago the suspended spans have been free of torsional vibrations of high amplitude.

Existing Concrete Slabs

The replacement of the concrete slabs of the existing roadway would be a costly undertaking and an annoyance to the users of both roadways for a period of two years or more. An Epoxy resurfacing is all that is required to put this pavement in excellent condition.

Grid Paving

It is customary to fill grids with concrete for their full depth. When so done the transverse rods are embedded and the concrete is more securely retained. With a roadway underneath we must recognize the hazard of chunks of concrete breaking loose and dropping on vehicles below if the sheet metal form rusts out.

The nine inch wide slots in the pavement should be omitted since through these openings water and debris may drop onto vehicles using the lower roadway.

Roadway Lighting

The writer doubts that handrail lighting has proven any efficiency over that obtained by lights elevated above the roadway and he is not aware of the employment of such lighting on any bridge.